A STUDY OF SURFACE SETTLEMENTS DUE TO EXCAVATIONS BY SHALLOW TUNELLING

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ABSTRACT

The Surface Settelments due to Excavations for Tehran's Metro is investigated by both precise measurement and analytical/numerical methods.The results of FEM Analysis using hyperbolic modelling varied widely from the measured values while some of the analytical relationships shows an better agreement with them. In Addition , the heaving effect of the work face and the transverse settlement trough is also analysed.

1. INTRODUTION

The first phase of construction of Tehran's Metro included a total length of 70 km, whereof 60 km should be constructed under the ground surface, 3 km in open cutting and the remainder over the ground surface.

The excavation were done in 40 km using a extra for local conditions prepared shield machine and in 20 km of the rest after NATM. Based upon the local subsoil conditions, it was improved to provide for excavation and supporting in 2 successive phases.

In the first phase, the upper half section was excavated at 0.8 to 1.2m, then a steel frame was installed immediately adjacent to the working face connected to its preceding, a 0.6 mm bar mat reinforcement between adjacent frames along the tunnel side wall and at the roof. The provisional lining is to be accomplished by putting a shotcrete facing over the reinforcement.

In the second phase the excavation and supporting of the lower half section was to be completed as before. At least the final lining as reinforced concrete shale must be done.

Surface settlement of loose ground due to excavating a shallow tunnel in urban areas may cause excavation related damages on buildings at the surface ,if exceeding the allowable limits.Therefore some research efforts were undertaken in order to predict the behaviour of the surrounding earth mass and the shape of settlement trough.

2. MEASURMENTS

In order to measure surface settlements, it was located a grid consisting of 70 settlement points above the tunnel at the study areas.

The measurement points above the tunnel alignment were placed close together accounting for higher sensivity to settlements there (see fig. 1).

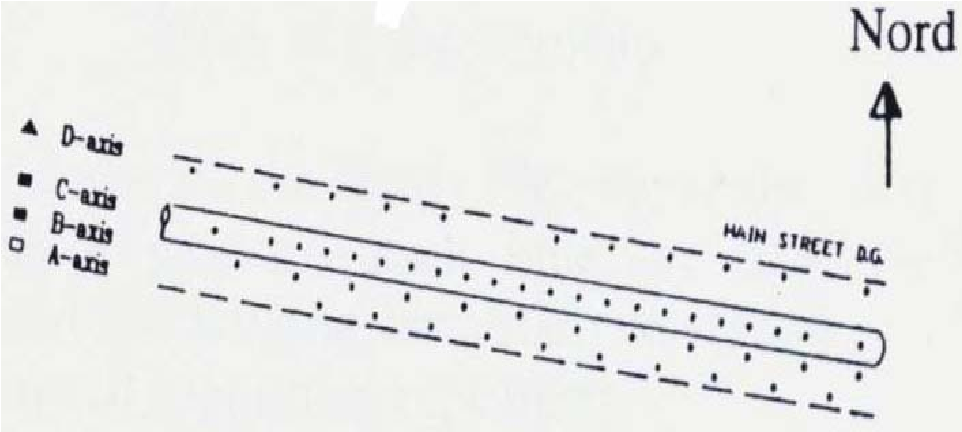


Figure 1

Grid of measurement points

The first set of survey was accomplished by precise levelling. The results of this surveying was taken as the baseline for the subsequent three measurements.

3. MODELLING SOIL BEHAVIOUR

The real behaviour of the soil is a elastic plastic one, especially for very high stresses. Under these conditions, in order to calculate stress and strain values, it is possible to use elastic non – linear soil models. One of these is the designated hyperbolic model.

The study of stress – strain curves obtained from 3 – axial tests for various types of soils indicated, that these approach a hyperbolic shape (Duncan & Chang, 1970). In non linear analysis, the hyperbolic relationship may be estimated using the parameter excavation, tangential Young – modulus and B, bulk - modulus, whereat the first parameter depends on surrounding earth pressure and the latter only depend on confined pressure. It is possible the three major soil characteristics, i. e. elastic, non – linear and stress – depending behaviour of the soil, changing the parameter E, and B together with varying the stresses within the soil using an step by step analyses.

The parameters in hyperbolic model may be obtained from a series of standard drained or unconsolidated undrained 3 – axial tests. For soils under natural conditions, the tests must be done only on undisturbed samples. The drainage conditions in test have to be the same as in the field. The discrepances in test results must be eliminated according to one's judgment, and the curves must be in hyperbolic φ adjusted properly. Since the mohr – circle envelopes to establish the values of c and values from various 3 – axial tests φ model may not be a straight forward line at interval under study, may then be obtained φ may be used and the value of $(\delta_\varepsilon / p_a)$ plotted the relationship versus log as follows:

$$(1)\ \varphi = \varphi_0 - \Delta\varphi \log_{10}(\delta_\varepsilon / p_a)$$

In order to estimate the parameter obtained from tests and to select the proper values for them in the case of non – existence of sufficient data base, there is tables suggested for various types of soils (Duncan et al, 1980). It is also suggested conservative values making an application of

hyperbolic modeling for compacted soils (embankments) possible but must be revised for use in natural soils.

For the soil in study area, with regard to impartibility to perform triaxial tests, the following values was selected for use in hyperbolic model:

$$n = 0.3, k = 1200, m = 0.15, k_b = 400, R_f = 0.7,$$

$$\varphi_0 = 0.6, c = 3.5, K_{ur} = 1500$$

4. NUMERICAL ANALYSIS USING FEM

In order to analysis stresses and establish values for strains in granular soils and fines under drained and undrained conditions, a computer program was developed based upon FEM. The FE – Mesh used consists of 46 tetraeder elements and 171 nodes (see fig. 2)

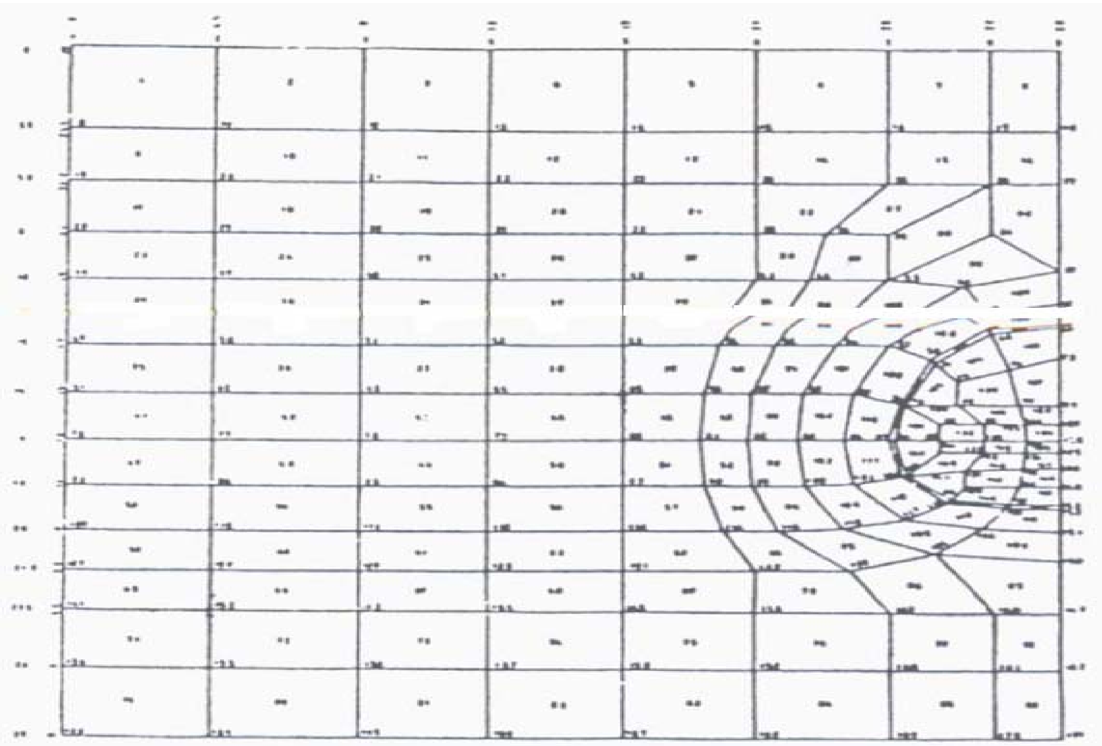


Figure 2

Mesh of finite elements

To make out the input file, the analysis was run in 4 phases, each time the data file adjusted on changing excavation conditions and provisional supporting repeatedly.

In the first phase, the changes in stress and strain in soil mass due to excavation of the upper half – section was analysed. In the second phase, the changes in stress and strain after bringing the provisional supporting in was computed. The 3. and 4. phases was such as the first 2 but only for the lower half section.

With regard to the results obtained from field surveys and the little amount of displacement, the analysis was run for linear model again.

5. EMPIRICAL APPROACHES

Up to this day several efforts was undertaken to describe the shape of the settlement trough mathematically, but it is uncontroversial, that a normal probabilistic curve is the best fitted for the predictive purposes. Here it was assumed. That the transverse settlement profile have the shape of normal or Gaussian probabilistic curve (see fig. 3)

According to empirical data obtained from full – scale tunnels, Peck (1969) have suggested the following relationship to describe the shape of the settlement distribution curve.

$$(2)\ S(x) = S_{\max} e^{2(\ )}$$

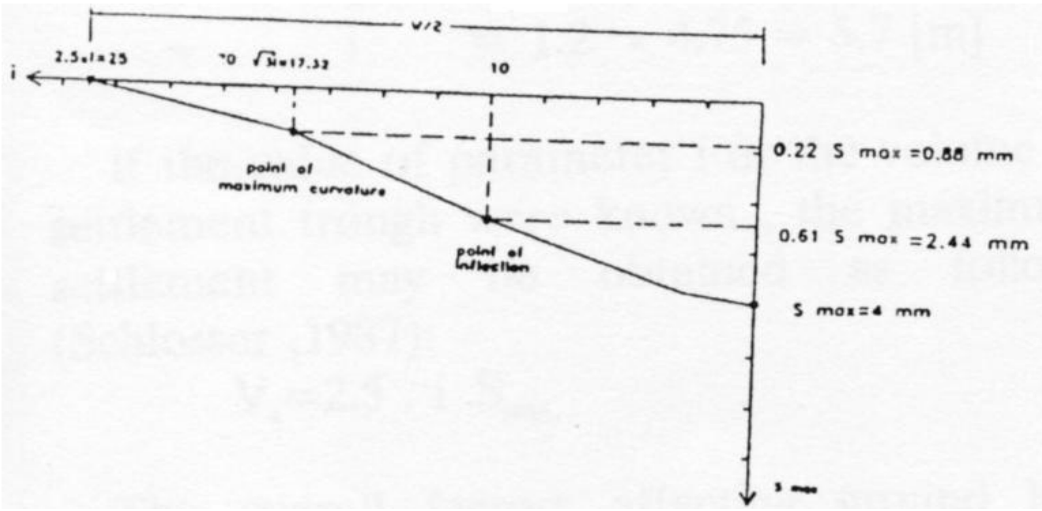


Figure 3

The shape of settlement trough (adjusted)

If the values of i and the maximum settlement were known, it possible to estimate the surface settlement at any desired point above the tunnel. Many efforts were undertaken to predict the value of i (the width of settlement trough parameter) experimentally from surveys performed in various types of underground. In Britain, O' Reilly & New (1982) have suggested based upon the case studies on full – scale tunnels, the following relationships:

For cohesive soils:

$$(3)\ \mathrm{i} = 0.43(\mathrm{z}_0 - \mathrm{z}) + 1.1[\mathrm{m}](3 \le \mathrm{z}_0 < 34)$$

For granular soils:

$$(4)\ \mathrm{i} = 0.28(\mathrm{z}_0 - \mathrm{z}) - 0.1[\mathrm{m}](6 \le \mathrm{z}_0 < 10)$$

Using proper parameters in this case the value of i may be obtained as follows:

$$z_0 = c + \frac{D}{2} = 22 + 4.75 = 15.75$$

$$z = 0$$

$$\Rightarrow i = 0.28(15.75) - 0.1 = 4.31[m]$$

The relationships obtained from model studies by Potts & Attkinson (1997) are similar to the relationships (3) and especially to (4):

$$(5)\ i = 0.25(c + D) = 0.25(z + 2R) \quad \text{for } \delta_s = 0$$

$$(6)\ i = 0.25(1.5c + D) = 0.25(1.5z + 2R) \quad \text{for } \delta_s \neq 0$$

Using adequate values for the parameters above, WE obtained for i:

$$i = 0.25(11 + 2 \times 4.75) = 5.1m$$

$$i = 0.25(1.5 \times 11 + 2 \times 4.75) = 6.5m$$

Attewell (1997) suggested using a quasi - empirical relationship based upon the stochastic theory of Litwiniszyn from 1956, the following approach:

$$i / a = \alpha[z / 2\alpha]^n$$

For the parameters and tunnel, it was suggested diverse values by different researchers as reported by Schlosser (1987) and summarized in the following table:

TABLE 1

Calculated values for parameter i from different authors

autor	α	**n**	**i**	**remarks**
		(m)		
Attewell(1997)	1	1	9.36	clay
Schmidt & Clough (1977)	1	0.8	8.17	
Sagaseta (1986)	1.5	1	10.75	

At least Peck (1969) have presented curves based upon results from measurements representing a relationship between the width of settlement trough and the tunnel depth under various underground soil conditions (see fig. 4)

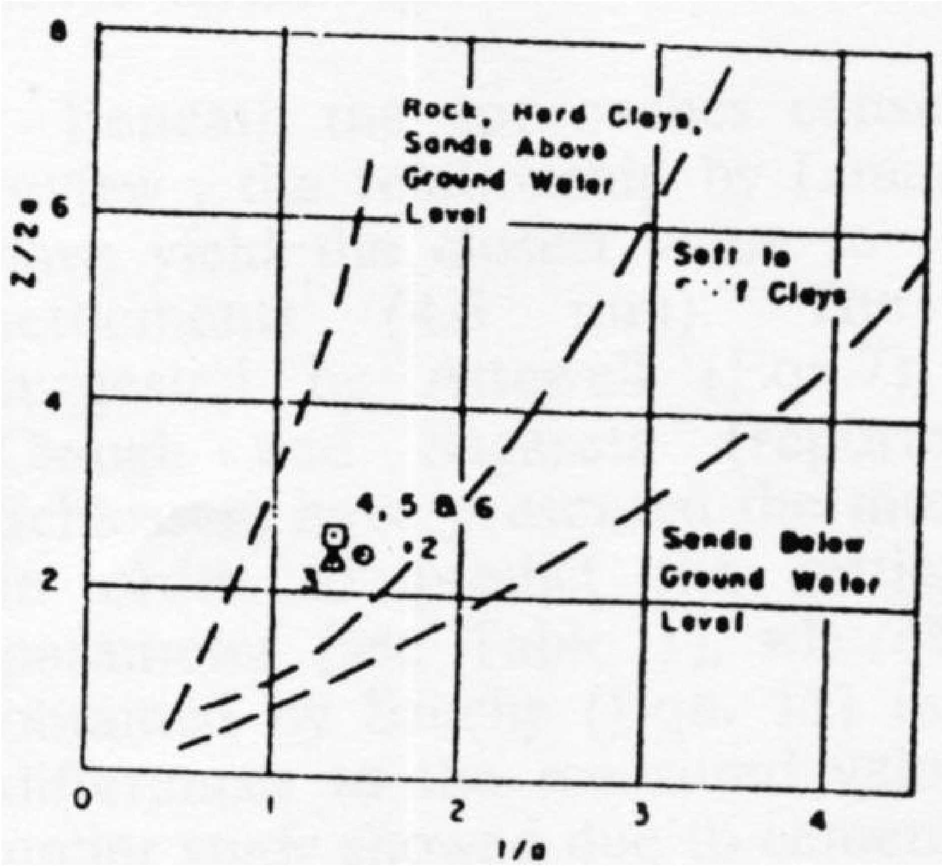


Figure 4

Relationship between trough width and depth of tunnel in different materials (Peck, 1969)

For tunnels under study this diagram gives the following values for the parameters i:

$$z/2a = 15.75/9.5 = 1.65$$

$$\Rightarrow i/R = 1.2$$

$$\Rightarrow i = 1.2 \times 4.75 = 5.7[m]$$

If the value of parameter i or the volume of settlement trough were known, the maximum settlement may be obtained as follows (Schlosser, 1987):

$$(8)\ V_s = 2.5.i.S_{max}$$

The overall factors affecting ground loss were analysed by Attewell (1977). On the other hand based upon experiences obtained from metro projects in Madrid and Caracas, Oteo & Sagaseta (1982) have suggested many improvements in order to include some factors such as Construction details using FE – Modelling. According to the curves obtained from their analysis, the maximum value of settlement trough may be obtained as follows:

$$(9)\ E.S_{max} / \gamma.D^2 = 0.85 - \nu$$

$$\text{for } \gamma = 2\left[t/m^3\right] \Rightarrow S_{max} = 5.9[mm]$$

6. THEORETICAL APPROACHES

To take in to consideration the mechanical behaviour of the soil mass, Limanov (1957) have presented his method based upon the theory of elasticity using probability function of Aversin, originally developed for shape of settlement curve in mining holes. He conducted based upon the Maxwell theorem, the deformations developed at the ground surface from the deformations developed, if the lining exercised a uniform pressure on the soil roundabout.

The Limanov's relationship for computing the maximum settlement is as follows:

$$(10)\ S_{max} = \left(1-\nu^2\right)\frac{P}{E}\left(4{r_0}^2 h_0 / {h_0}^2 - {r_0}^2\right)$$

Szechy (1977) have also suggested the following approach for non – cohesive soils:

$$S_{max} = 3\pi r^2 \tan\alpha / 4\tan^2\beta\left[r(1+\cos ec\beta)\right]$$

$$(11) + H\left(2 + \cos ec\beta \frac{H}{r}\right)$$

It results for given conditions:

$$\left(H = 11[m], \beta = 45 - \frac{40}{2} = 25, \phi = 40^\circ, r = 4.75[m]\right)$$

He also obtained upon the theory of $S_{max} = 23.8[mm]$A maximum settlement of Protodiakonov regarding the hight of loosened area for the width of settlement trough the following approach:

$$(12)\ W = 2\left[(r+h)\tan\beta + r\sec\beta + r\cot an\phi\right]$$

In this case, it will be obtained for the width of settlement trough

$$W = 28.2[m]$$

7. CONCLUSION

The values obtained for the maximum settlement using FEM indicated appreciable differences to the measurements (see fig. 5)

One of the reasons for this ids the small values taken for the elastic modulus. With regard to the massiveness and hardness of the soil in the study area, we may be take Excavation – modulus greater than values obtained from linear model analysis and the little amount of strains, it may be concluded that the behaviour under loading lies in linear region.

Beneath the approaches considered in this review, the relationship by Limanov (Eqn.10) have yield the closest value to the measured settlement (4.8 mm). the approaches suggested by Attewell (Eqn.7), Schmidt & Clough and Sagaseta (reported both by schlosser) have presented the most real results in order to predict the settlement trough parameter (see Table 1), whereas the results obtained by Szechy (Eqn.11) indicated great differences to the measured values. The soil under study ahowed due to cementization effect high cohesivness and thereafter the Szechy approach suited for granular soils originally, yield only un appropriate outcomes.

The most important fact observed may be summerized as follows:

1) the settlements due to excavation of upper half – section makes 50 percent of total settlements observed. It is very important here to do it precise with excavation and putting on supports.

Transverse settlement trough; a comparision between measurement and prediction

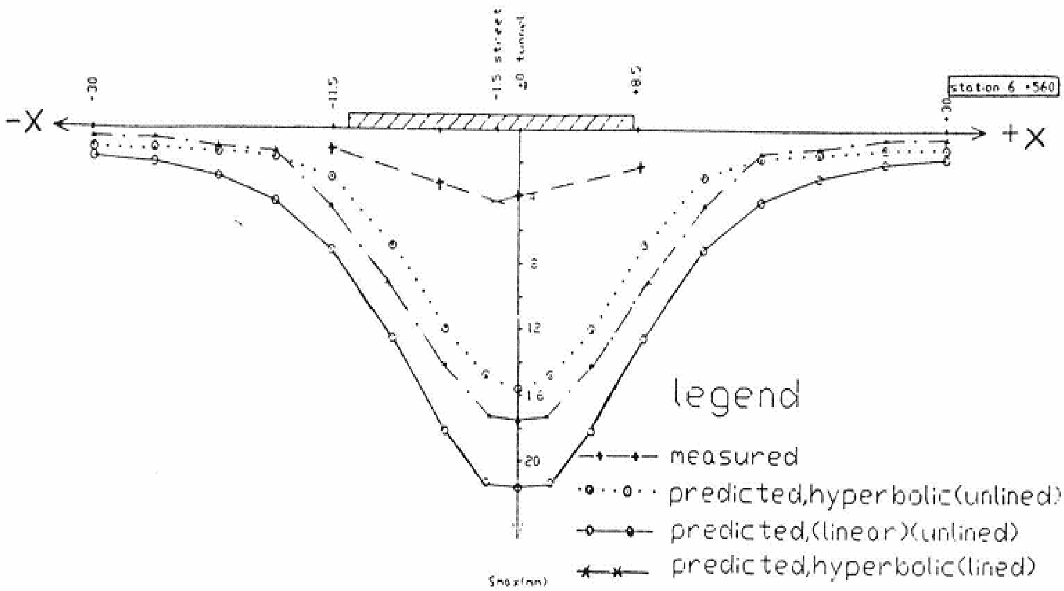


Figure 5

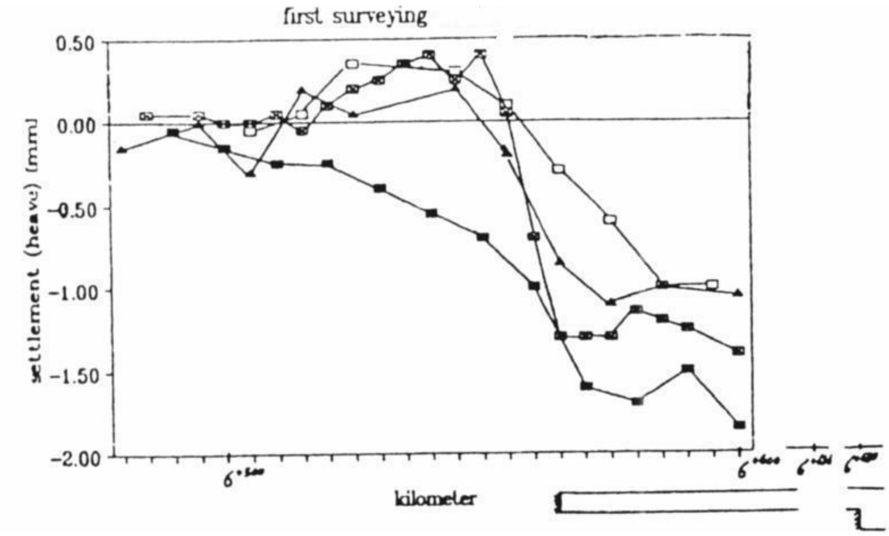
Transverse settlement trough : A comparision between measurement and prediction

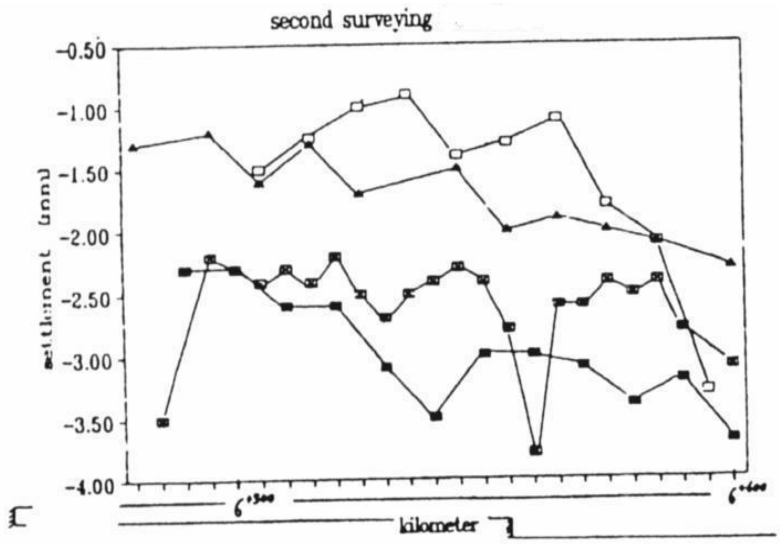
2) As reported before by another authors. It observed analysis heaving at the surface directly above the excavation face (see fig. 6).

In general the increasing water pressure due to interrupting seewage pipe lines and injection pressure may play here analysis certain role but in this case with regard to the high permissibility of the soil, no water entrance was observed through supporting nor the likely reason for heaving up the ground surface may have been an excavation of the soil due to shearing.

In compacted granular soils in excess of to the friction, there is analysis considerable grain locking. By exercising a shear force. The amount of grain locking decreases gradually. One grain rolls over the other. With respect to the high compaction of the soil at the study area. This condition leads to the increasing of the soil volume and therefor to its expansion. The wide range of displacements (up to 100 maximum from the tunnel axis) is also to be expectable with regard to the high compaction of the soil and its function as analysis uniformly rigid mass.

3) the maximum settlement was observed not at the beginning of the tunnel alignment, but along the street axis above. It may be resulted from static and dynamic loading due to traffic. The arching of the soil above the tunnel axis may confine the displacement there and increase the settlements at the adjacent points.





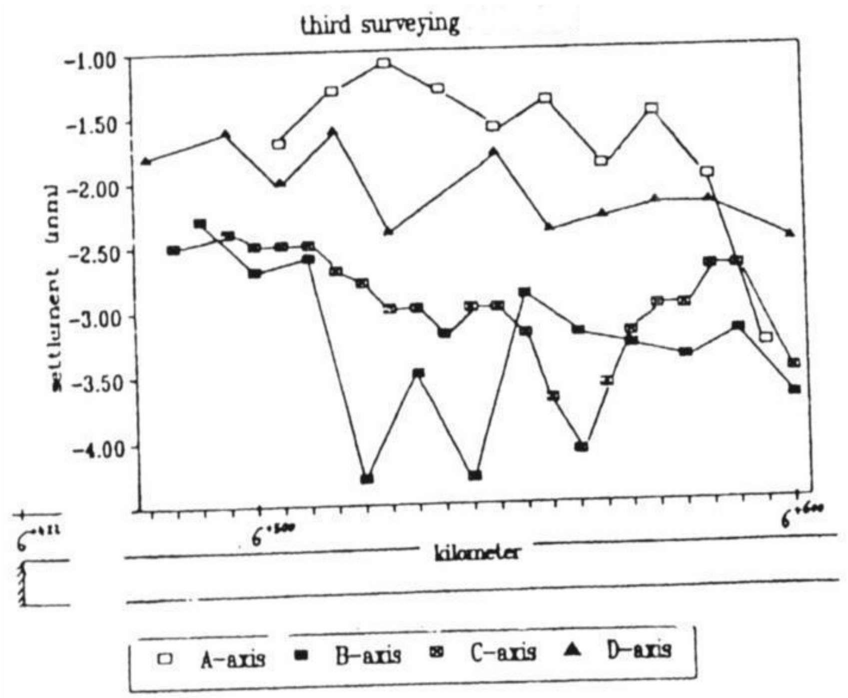


Figure 6

Longitudinal settlement profiles a comparision at different excavation stages

ABRREVIATIONS

$a, D/2, R, r, r_0 =$ tunnel radius

c = cohesion

E = module of elasticity

$h_0, x_o =$ distance ground surface – tunnel axis

H = soil cover

i = settlement trough parameter

$k, k_0, k_{ur}, m, n =$ Hyperbolic model constants

P = uniformly internal pressure

$P_a =$ atmospheric pressure

S(x) = settlement at point x

$S_{max} =$ maximum settlement

V_s = settlement trough volume

W = width of settlement trough

α = angle of soil extrusion

β = angle failure – plane to vertical plane

ν = poissons ratio

δ_3 = confined pressure

φ = angle of internal friction

φ φ_0 = by 1 atmosphere

$\Delta\varphi$ = variation of friction pro cycle

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